Field Investigation Findings Final Geotechnical Report for Residuals Collection and Treatment Facilities

Washington Aqueduct, Washington, D.C.

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CH2MHILL

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SECTION 1

Introduction

1.1 Scope of Geotechnical Report

This geotechnical report presents the results of Phase I and II subsurface investigations and basis of geotechnical design of the foundation systems and pavement design for the project. Additionally, the report presents the recommended geotechnical design strength parameters of soil and rock, seismic site classification, design groundwater elevations, lateral earth pressure, excavation support systems, trench excavation and backfill, site fill, micropile testing, and drilled shaft construction considerations.

1.2 Site Descriptions and Proposed Construction

The project site is located in the neighboring area between Maryland and northwest Washington, District of Columbia (D. C.), as shown in the site location map in Appendix A, *Site Location Map and Layout Plans*. The proposed constructions in this project are located at the Dalecarlia Water Treatment Plant (WTP), and near Dalecarlia and Georgetown Reservoirs, as shown in various site layout plans in Appendix A, *Site Location Map and Layout Plans*.

The following works and facilities are proposed in this project.

1. Forebay Dredge
2. Forebay Residuals Pump Station
3. Booster Control Station
4. Forebay Residuals Transfer Pipeline
5. Thickener Overflow Recycle Pipeline
6. Dalecarlia Sedimentation Basins 1 through 4
7. Dalecarlia Residuals Pump Station
8. Dalecarlia WTP Yard Piping/Yard Electrical/Site Civil
9. Georgetown Dredges
10. Georgetown Residuals Pump Station
11. Georgetown Residuals Transfer Pipeline in Georgetown Conduit
12. Residuals Processing Facility
13. Truck Wash
14. Residuals Processing Facility Yard Piping/Yard Electrical/Site Civil

SECTION 2

Regional Geology

The project site is located in the Potomac Terrain of the Piedmont Physiographic Province. The mountains of the Blue Ridge Province bound the Piedmont in the west and the Fall Zone, which separates the Piedmont from the Coastal Plain Province, bounds it in the east. The city of Washington D. C. sits on the Fall Zone. The Piedmont Physiographic Province in this area is divided into a lowland section and an upland section. The western lowland section, the Fredrick Valley, is formed on early Paleozoic limestone and dolomite. This section of the province also exhibits deposits of Triassic sandstone, siltstone and shale. The eastern upland section, the Potomac Terrain, consists of late Proterozoic to middle Paleozoic metamorphic and igneous rock. The Piedmont is characterized by gently rolling topography, deeply weathered bedrock, and relatively few solid outcrops. Rocks are strongly weathered in the Piedmont's humid climate, and bedrock is generally covered by a 6- to 60-foot thick saprolite blanket. Outcrops are commonly restricted to stream valleys, where erosion has removed saprolite.

The rocks underlying the project site have been identified as part of the Sykesville Formation of the Lower Cambrian period. The Sykesville Formation consists primarily of metamorphic rocks. The rocks resemble granite and granite gneiss, and were known in the first half of the twentieth century as the Sykesville Granite. According to the geologic map published as a part of the report titled "*Geology of the Chesapeake and Ohio Canal, National Historical Park and Potomac River Corridor, District of Columbia, Maryland, West Virginia, and Virginia*," the Sykesville Formation consists of gray quartzofeldspathic matrix with fragments and bodies of metamorphosed sedimentary, volcanic, and igneous rocks. Toward Washington D. C., intrusive igneous rocks of the Georgetown and Dalecarlia suites are visible at the surface.

Residual soils are present in the Piedmont Physiographic Province (Obermeier and Langer, 1986). Residual soils have developed from in-place weathering of underlying bedrock. They are distinguished from other naturally occurring soils (such as alluvia) in that they have not been transported and re-deposited. Chemical action has weathered or altered some of the minerals in the parent rock and the products consist of clay minerals, hydrous micas, and iron oxides (Sowers and Richardson, 1983). However, the residual soils retain the parent rock's mineral segregation, mineral alignment, and structural defects. The residual soils in this project site range from sand to silty or clayey sand to sandy gravel to sandy or clayey silt to sandy or silty clay and correspond to USCS groups SW, SP, SM, SC, GP, ML, and CL.

The degree of weathering decreases with depth, so that the boundary between soil and rock is gradual and difficult to identify. The degree of weathering is also laterally variable, resulting in an irregular top-of-rock surface (Obermeier and Langer, 1986).

Residual soils have a typical weathering profile. The weathered profile can be divided into four categories or zones with these general characteristics:

1. The **Surface Zone** consists of completely weathered soil displaying well-developed pedological horizons. The material has generally lost its visible remnant structure.
2. **Intermediate Zone** material has been weathered to soil-like consistency. These soils usually retain some of the parent rock's remnant structure, such as compositional banding and jointing. Soils derived from crystalline igneous and metamorphic rock, such as those at this site, are referred to as "saprolite." Soils derived from the weathering of sedimentary rocks are referred to as "residuum." Standard penetration test (SPT) results are highly variable in this material and can range up to 100 blows-per-foot (bpf). Soil excavation techniques can typically be used in this zone.
3. The **Partially Weathered Zone** is a transition between the residual soil and the intact bedrock and ranges from soil-like to rock-like. Inward weathering from joints results in boulder-like masses or in alternating soft and hard zones. Typically, SPT results are greater than 100 bpf, and auger borings may reach refusal within this zone. It may become necessary to change from soil-excavation techniques to rock-excavation techniques when excavating in this zone. However, this partially weathered zone material may also be rippable.

The **Bedrock Zone** is moderately weathered to fresh parent-rock material. Augers cannot penetrate it, and excavation typically requires drilling, blasting, or mechanical means.

SECTION 3

Subsurface Investigations

3.1 Review of Available Geotechnical Data

The following recent subsurface investigations were carried out in the vicinity of the Residual Processing Facility, Forebay, Dalecarlia Sedimentation Basins, and Georgetown Reservoir. Results from these investigations, including laboratory testing data, are presented in Appendix B, *Data from Historical Subsurface Investigations*.

1. Borings at Forebay, Sedimentation Basins, and Georgetown Reservoir by U. S. Army Corps of Engineers dated from May 1995 to October 1995.
2. Soil Boring Information for East Dalecarlia Processing Site by U. S. Army Corps of Engineers dated from February 2005 to March 2005.

The above subsurface investigation results showed that the soil profiles in the explored areas consist of fill (silty sand, sandy silt, and silty clay), residual soils (silty sand), partially weathered rock (highly weathered and decomposed rock), and bedrock (moderately weathered to unweathered gneiss) in a descending order. The depth to the top of rock ranges from 25 to 68 feet below the ground surface at the time of drilling. Recovery (REC) of rock cores ranged between 0 and 100 percent, with a representative value of 80 percent. Rock quality designation (RQD) ranged between 0 and 95 percent, with a representative average value of 40 percent. Unconfined compressive strength of the intact rock cores ranged from 2,404 to 9,039 pounds-per-square-inch (psi). Split tensile strength of the rock varied from 927 to 1,210 psi. Due to a limited number of samples tested, strength values higher or lower than those indicated above can be anticipated.

3.2 Phase I Subsurface Investigation

Phase I subsurface investigation program, including 9 borings as shown in Table 3-1, *Phase I Borings Summary*, was performed in August 2006. The boring location plans are presented in Appendix C, *Boring Location Plans*. Froehling & Robertson, Inc. (F&R) performed the subsurface investigation, which was inspected on a full time by a CH2M HILL geotechnical engineer. The borings were drilled using an ATV-mounted CME-55 drill rig, extending to a depth between 20 and 52 feet below the existing grade. The boring logs are presented in Appendix D, *Boring Logs*.

Table 3-1 Phase I Borings Summary

Boring	Depth (ft)	Location
FB-1 and FPS-1	33 to 33.2	Forebay area
PS-1	44.1	Dalecarlia Sedimentation Basins
GCP-1, GCP-2, GCP-3, GCP-4, GPS-1, and GPS-2	20 to 50	Georgetown Reservoir area

Generally, continuous-flight, hollow-stem augers with an inside diameter (ID) of 2-1/4 inches or 3-1/4 inches were used to advance the borings to the completion depth or auger refusal. Below the depth of auger refusal, rock coring techniques were used to advance borings to the planned finished depths. Standard Penetration Tests (SPTs) were performed using a standard 2-inch outside diameter (OD) split-spoon sampler driven 18 inches with a standard 140-pound hammer falling a distance of 30 inches, with rope and cathead, in accordance with ASTM D1586. The number of blows was recorded for each 6-inch increment, and the total number of blows required to drive the split-spoon sampler for the second and third 6-inch increments represents the SPT resistance, or the N-value. Disturbed soil samples were retrieved using the split-spoon sampler continuously in the top 10 feet and at a 5-ft interval thereafter. Two undisturbed Shelby tube samples were retrieved from borings GPS-2 and GCP-4 where cohesive soils were encountered.

Once auger refusal was encountered, rock coring was carried out using NQ-sized double-tube core barrel with wire-line to retrieve rock cores. REC and RQD values of each core run were recorded in the field. Rock cores were stored in wooden core boxes and photographs of the core boxes were taken before F&R transported the core boxes back to the laboratory.

Groundwater was recorded when encountered during drilling, at the completion of boring, and 24 hours or more after completion of boring. Additionally, two temporary observation wells were installed in borings GPS-2 and FPS-1 for long-term monitoring of the groundwater levels.

3.3 Phase I Laboratory Testing

Selected soil samples from Phase I Subsurface Investigation were tested for their index properties (Atterberg Limits, natural moisture contents, and gradations), organic content, corrosivity, and shear strength properties. Selected intact rock core samples were tested for their unconfined compressive strength. The numbers of various tests performed are summarized in Table 3-2, *Summary of Laboratory Tests Performed in Phase I Subsurface Investigation*, with their ASTM Standards. Laboratory testing results are presented in Appendix E, *Laboratory Testing Results*. The results of historical and Phase I unconfined compressive strength (UCS) tests are summarized in Table 3-3, *Summary of Laboratory Unconfined Compressive Strength of Intact Rock from Recent and Phase I Subsurface Investigation*. Additionally, the photographs of core boxes and core samples after unconfined compressive strength tests are presented in Appendix F, *Photographs of Core Boxes and Core Samples after Unconfined Compressive Strength Tests*.

Table 3-2 Summary of Laboratory Tests Performed in Phase I Subsurface Investigation

Number of Tests Performed	Type of Test	ASTM Standard
5	Unconfined Compressive Strength	ASTM D 2938
4	Sieve Analysis	ASTM D 422
1	Hydrometer	ASTM D 422
6	Water Content	ASTM D 2216
3	Atterberg Limits	ASTM D 4318
3	Corrosivity	ASTM D4972, D4230, D1125, D512
2	Organic Contents	ASTM D2974
1	Consolidated-undrained Triaxial Test with Pore Water Pressure Measurements	ASTM D4767

Table 3-3 Summary of Laboratory Unconfined Compressive Strength of Intact Rock from Recent and Phase I Subsurface Investigation

Boring	Depth (ft)	q_u (psi)
WA1	33	4,607
WA2	54	2,404
WA5	34	6,639
WA8	59	9,039
GPS1	40	10,100
GPS2	42	4,450
FB1	30	6,600
PS1	29	6,630

3.4 Phase II Subsurface Investigation

Phase II subsurface investigation program, consisted of 20 borings as shown in Table 3-4, *Phase II Borings Summary*, and was executed between February and March 2007. The boring location plans are presented in Appendix C, *Boring Location Plans*. F&R performed the subsurface investigation, which was inspected on a full time either by a CH2M HILL geotechnical engineer or a geologist. The borings were drilled using either an ATV-mounted or truck mounted CME-55 drill rig, extending to a depth between 15 and 88.4 feet below ground surface (bgs). The boring logs are presented in Appendix D, *Boring Logs*.

Table 3-4 Phase II Borings Summary

Boring	Depth (ft)	Location
BH1, BH2, …, BH16 (total 16 borings)	39.6 to 88.4	Residuals Processing Facility Site
GCP5, GCP6, GPS3, and GPS4	15 to 48.6	Georgetown Reservoir area

Generally, continuous-flight, hollow-stem augers with an inside diameter (ID) of 2-1/4 inches or 3-1/4 inches were used to advance the borings to the completion depth or auger refusal. Below the depth of auger refusal, rock coring techniques were used to advance borings to the planned finished depths. Standard Penetration Tests (SPTs) were performed using a standard 2-inch outside diameter (OD) split-spoon sampler driven 18 inches with a standard 140-pound hammer falling a distance of 30 inches, with rope and cathead, in accordance with ASTM D1586. The number of blows was recorded for each 6-inch increment, and the total number of blows required to drive the split-spoon sampler for the second and third 6-inch increments represents the SPT resistance, or the N-value. Disturbed soil samples were retrieved using the split-spoon sampler continuously in the top 10 feet and at a 5-foot interval thereafter except GCP-6 where SPT was performed continuously in the top 20 feet and at a 5-foot interval thereafter. “Undisturbed” Shelby tube samples were retrieved from borings GCP-6 for further laboratory testing.

For the borings at the Residuals Processing Facility site, Unexploded Ordnance (UXO) check was performed every 1 foot from 0 to 10 feet bgs and at 13.5, 18.5, and 23.5 feet bgs. Due to the check for UXO, split spoon was sampled every 1 foot for top 10 feet. Additionally, Photo Ionization Detector (PID) was used to detect potential volatile organic constituents (VOC) and no elevated PID reading was observed.

Once auger refusal was encountered, rock coring was carried out using NQ-sized double-tube core barrel with wire-line to retrieve rock cores. REC and RQD values of each core run were recorded in the field. Rock cores were stored in wooden core boxes and photographs of the core boxes were taken before F&R transported the core boxes back to the laboratory.

Boulders were encountered at residuals processing facility site which was manifested by auger refusal at shallow depth, grinding sound at shallow depth, and rock coring through boulders. The approximate depth and thickness of possible or confirmed boulders at the residuals processing facility site are summarized and tabulated in Table 3-5, *Possible Boulders Encountered during Soil Boring*.

Table 3-5 Possible Boulders Encountered during Soil Boring

Boring	Approximate Top Depth of Boulder (feet)	Approximate Top Elevation of Boulder (feet)	Approximate Thickness of Boulder(s) (feet)	Presence Possibility of Boulder	Identification Method
BH-02	12.5 - 18	205.4 - 199.9	13	Confirmed	Borehole offsets 3 times and auger refusal at 12.5, 14.5, 14.6, and 18 feet bgs. The boulder was cored through using rock core.
BH-03	14 - 18	199.3 – 195.3	5 or thicker	Confirmed	Borehole offsets 4 times and auger refusal at 14, 14.5, 17.5, 17.5 and 18 feet bgs. The boulder was cored through using rock core.
BH-04	18.5	202	-	Possible	Grinding sound of auger
BH-05	12 – 17.5	207.4 - 201.9	6 *	Confirmed	Borehole offsets 3 times and auger refusal at 16, 12, and 17.5 feet bgs within 3 feet from original location.
BH-07	10	212.1	3*	Possible	Grinding sound of auger from 10 to 13 feet
BH-08	6.5	214.7	5.5*	Possible	Grinding sound of auger from 6.5 to 12 feet
BH-10	6	217.8	-	Possible	Grinding sound of auger
BH-11	10	213.3	-	Possible	Grinding sound of auger
BH-12	9	213.1	-	Possible	Grinding sound of auger
BH-13	17.8	204.5	-	Confirmed	Auger refusal at 17.8 feet bgs and borehole offsets 1 time.

Note: '*' indicates that the thickness of boulder is not certain but estimated using best judgment.

Nine geoprobes were advanced at the residuals processing facility site to obtain samples for environmental screenings. Six out of nine probing encountered refusals at a relatively shallower depth ranging from 14.7 feet to 31.7 feet bgs. Boulders might be encountered at these refusal depths.

Groundwater was recorded when encountered during drilling, at the completion of boring, and 24 hours or more after completion of boring. Additionally, two temporary observation wells were installed in borings BH-05 and GCP-5 for long-term monitoring of the groundwater levels.

3.5 Phase II Laboratory Testing

Selected soil samples from Phase II Subsurface Investigation were tested for their index properties (Atterberg Limits, natural moisture contents, and gradations), organic content, corrosivity, and shear strength properties. Selected intact rock core samples were tested for their unconfined compressive strength. The numbers of various tests performed are summarized in Table 3-6, *Summary of Laboratory Tests Performed in the Phase II Subsurface Investigation*. Laboratory testing results are presented in Appendix E, *Laboratory Testing Results*. The results of all unconfined compressive strength tests performed in phase II are summarized in Table 3-7, *Summary of Laboratory Unconfined Compressive Strength of Intact Rock from Phase II Subsurface Investigation*. Additionally, the photographs of core boxes and core samples after unconfined compressive strength tests are presented in Appendix F, *Photographs of Core Boxes and Core Samples after Unconfined Compressive Strength Tests*.

Table 3-6 Summary of Laboratory Tests Performed in the Phase II Subsurface Investigation

Number of Tests Performed	Type of Test	ASTM Standard
16	Unconfined Compressive Strength	ASTM D 2938
18	Sieve Analysis	ASTM D 422
6	Hydrometer	ASTM D 422
9	Water Content	ASTM D 2216
10	Atterberg Limits	ASTM D 4318
7	Corrosivity	ASTM D4972, D4230, D1125, D512
3	Organic Contents	ASTM D 2974
1	Consolidated-undrained Triaxial Test with Pore Water Pressure Measurements	ASTM D 4767
3	California Bearing Ratio	ASTM D 1883

Table 3-7 Summary of Laboratory Unconfined Compressive Strength of Intact Rock from Phase II Subsurface Investigation

Boring	Depth (ft)	q_u (psi)
BH-01	41	5203
BH-03	17	12979
BH-04	66.3	12916
BH-05	71	7962
BH-05	75	6161
BH-06	67	7136
BH-07	70.2	6779
BH-08	75.3	7575
BH-09	66.3	10663

Boring	Depth (ft)	q_u (psi)
BH-10	63	3520
BH-11	75.8	7548
BH-13	58	7470
BH-14	64	4888
BH-15	49	6522
BH-16	52	4048
BH-16	65.5	8868

CBR test results are summarized in Table 3-8, Summary of CBR at 0.1 inch Penetration. ASTM D698 was used to determine moisture-density relationship for these CBR tests.

Table 3-8 Summary of CBR at 0.1 inch Penetration

Sample	Depth (ft)	CBR @ 90% Compaction	CBR @ 95% Compaction	CBR @ 100% Compaction
BH-01	7-15	2.4	2.8	4.4
BH-06	5-15	3.2	2.8	7.2
BH-14	5-15	2.0	3.2	4.0

SECTION 4

Subsurface Conditions

4.1 Subsurface Profiles at Residuals Processing Facility Site

Based on the soil boring and rock coring logs of phase II subsurface investigations, the subsurface conditions at the residuals processing facility consist of the following strata, in a descending order starting from the ground surface:

- Fill, consisting of clayey/sandy silt, silt, silty/clayey sand, sand, clay, gravel, boulders, brick, and concrete debris;
- Residual soils, consisting of sandy silt and silty sand;
- Partially weathered rock; and
- Bedrock, consisting of moderately weathered to unweathered rock.

It should be noted that the descriptions of the subsurface conditions in the following sections were based on available subsurface investigations and variations should be expected. Detailed characteristics of the stratification with thickness less than the sampling interval of SPTs could not be detected from the soil borings and were not included in the general subsurface stratification presented in this report. Also, it should be noted that subsurface stratification generally changes gradually, while distinct breaks were used in the boring logs to represent stratum change.

4.1.1 Fill

Fill, typically consisting of brown clayey/sandy silt, silt, silty/clayey sand, sand, clay, and gravel, was encountered in all phase II boring locations in residuals processing facility site. The thickness of the fill layer varied between 13.5 and 38.5 feet. Approximate N-values of the top 10 feet fill were estimated based on 2 consecutive 1-foot sampling, which was due to UXO check. Generally, the SPT N values in the Fill vary from 3 to 60 blows-per-foot (bpf), with an average value of 15 bpf. However, the SPT N value may be higher than 50 blows per 6 inches when boulders or concrete debris were encountered during SPT testing. According to the correlation by Peck et al. (1974), the friction angle of gravel and clayey/silty sand ranges from 28 to 40 degrees, with an average representative value of 33 degrees. The undrained shear strength of silt and clay, according to hand pocket penetrometer readings and the correlation by Bowles (1988) using SPT N values presented in EM 1110-1-1905, varies from 500 to 2500 pounds-per-square-foot (psf).

In this fill layer, boulders, concrete, wood chips, asphalt, slag, and bricks were randomly distributed at the residual processing facility site. Boulder(s) with thickness of 13 feet were cored through in the boring BH-02. Additionally, driller reported that steel debris were recovered among the soil cuttings in one of the sixteen borings.

4.1.2 Residual Soils

Residual soils were encountered below the fill. The thickness of this stratum ranged from 5 to 25 feet, with an average thickness of 13 feet. The residual soils consist primarily of brown, medium dense to dense silty sand or clayey sand, and brown, stiff to very stiff sandy silt. SPT N-values in this stratum ranged from 8 to 76 bpf, with an average value of 30 bpf. The friction angle of the soils in this stratum ranged from 30 to 41 degrees, with an average value of 35 degrees, according to the correlation by Peck et al. (1974).

4.1.3 Partially Weathered Rock

Partially weathered rock underlies the residual soil stratum and extends to the top of bedrock. This stratum ranged in thickness from 12.5 to 44 feet, with an average thickness of 24 feet. The degree of weathering of the rocks in this stratum is highly to extremely weathered and the recovered samples from split spoon can be classified as silty sand or sandy silt with variable amounts of decomposed rock fragments. All SPT N-values in this stratum were greater than 50 blows per 6-inch increment, which is the definition of SPT refusal. The SPT refusal was used to differentiate partially weathered rock from residual soils.

4.1.4 Bedrock

Bedrock was encountered below the weathered rock and often identified by achieving auger refusal. Occasionally, the first few core runs below the auger refusal had very low REC and RQD values. In these cases, the bedrock was considered to be below the auger refusal where reasonable REC and RQD values could be obtained. The bedrock encountered at this site is mostly moderately weathered to unweathered gneiss. The bedrock is generally considered to be medium soft to hard. However, there are some isolated, extremely fractured zones where the rock can be classified as soft. The depth to the top of bedrock varied from 39.6 to 71 feet below the ground surface at the time of drilling. Generally, the elevation of bedrock encountered ranged from 148 to 178 feet. The REC in the bedrock ranged from 40 to 100 percent, with an average value of 95 percent. The RQD of the bedrock ranged from 17 to 100 percent, with an average value of 60 percent.

Unconfined compressive strength (UCS) of sixteen intact rock core samples tested in the Phase II Subsurface Investigation ranged from 3,520 to 12,979 psi, with an average value of 7,500 psi. The recent subsurface investigation provided four unconfined compressive strength tests with UCS values of 2,404 and 4,607 psi for weathered gneiss and 6,639 and 9,039 psi for unweathered gneiss. It should be noted that the unconfined compressive strength of intact rock core samples may be greater than the compressive strength of in-situ rock mass with poor RQD. Splitting tensile strength tests on three rock core samples in the recent subsurface investigation showed that the tensile strength ranged from 927 to 1,210 psi, with an average value of 1000 psi.

4.2 Subsurface Profiles at Georgetown Reservoir, Forebay, and Dalecarlia Water Treatment Plant

Based on the soil boring and rock coring logs of Phase I and II subsurface investigations, the general subsurface profile at Georgetown reservoir, Forebay area, and Dalecarlia water treatment plant consist of the following strata, in a descending order from the ground surface:

- Fill, consisting of gravel, clayey sand, silt, and sandy clay;
- Residual soils, consisting of silt, sand, and silty sand;
- Partially weathered rock; and
- Bedrock, consisting of moderately weathered to unweathered rock.

It should be noted that the descriptions of the subsurface conditions in the following sections were based on available subsurface investigations and variations should be expected. Detailed characteristics of the stratification with thickness less than the sampling interval of SPTs could not be detected from the soil borings and were not included in the general subsurface stratification presented in this report. Also, it should be noted that subsurface stratification generally changes gradually, while distinct breaks were used in the boring logs to represent stratum change.

4.2.1 Fill

Fill, typically consisting of gray gravel, clayey/silty sand, silt, and sandy clay, was encountered at some of the boring locations. Boulders, wood, and brick fragments were observed in the recent borings at Dalecarlia Water Treatment Plant. The thickness of the fill layer varied between 0 and 18.5 feet. SPT N-values in the Fill vary from 4 to 54 blows-per-foot (bpf), with an average value of 22 bpf. According to the correlation by Peck et al. (1974), the friction angle of gravel and clayey/silty sand ranges from 28 to 40 degrees, with an average representative value of 33 degrees. The undrained shear strength of silt and clay, based on hand pocket penetrometer readings and the correlation by Bowles (1988) using SPT N values presented in EM 1110-1-1905, varies from 500 to 2500 pounds-per-square-foot (psf).

4.2.2 Residual Soils

Residual soils were encountered below the fill. The thickness of this stratum ranged from 11 to 23 feet, with an average thickness of 17 feet. The residual soils consist primarily of brown, medium dense, silty sand or clayey sand and brown, stiff to very stiff, sandy silt. The presence of cobbles and boulders in this layer were also expected based on the grinding sound of steel auger during soil boring. SPT N-values in this stratum ranged from 5 to 47 bpf, with an average value of 20 bpf. The friction angle of the soils in this stratum ranged from 30 to 39 degrees, with an average value of 33 degrees, according to the correlation by Peck et al. (1974).

4.2.3 Partially Weathered Rock

Partially weathered rock underlies the residual soil stratum and extends to the top of bedrock. This stratum ranged in thickness from 0 to 16.5 feet, with an average thickness of 9 feet. The degree of weathering of the rocks in this stratum is highly to extremely weathered and they can be classified as silty sand with variable amounts of decomposed rock fragments. All SPT N-values in this stratum were greater than 50 blows per 6-inch increment or less, which is the definition of SPT refusal.

4.2.4 Bedrock

Bedrock was encountered below the weathered rock and often identified by achieving auger refusal. Occasionally, the first few core runs below the auger refusal had very low REC and RQD values. In these cases, the bedrock was considered to be below the auger refusal where reasonable REC and RQD values could be obtained. The bedrock encountered at this site is mostly moderately weathered to unweathered gneiss. The bedrock is generally considered to be medium soft to hard. However, there are some isolated, extremely fractured zones where the rock can be classified as soft. The depth to the top of bedrock varied from 23 to 50 feet below the ground surface at the time of drilling. Generally, the elevation of bedrock encountered ranged from 98 to 134 feet. The REC in the bedrock ranged from 83 to 100 percent, with an average value of 94 percent. The RQD of the bedrock ranged from 32 to 95 percent, with an average value of 66 percent. Please note, REC and RQD as low as 38 percent and 0 percent, respectively were recorded in the recent soil borings.

Unconfined compressive strength of four intact rock core samples tested in the Phase I Subsurface Investigation ranged from 4,450 to 10,100 psi, with an average value of 6,900 psi. It should be noted that the unconfined compressive strength of intact rock core samples may be greater than the compressive strength of in-situ rock mass with poor RQD.

4.3 Soil/Rock Properties

Based on Phase I and II subsurface investigations and recent investigation data, the strength parameters of the soil and rock strata at Residual Processing Facility, Forebay, Georgetown Reservoir, and Dalecarlia WTP were estimated and presented in Table 4-1, *Summary of Soil and Rock Strength Parameters at Residuals Processing Facility Site*. The strength parameters of the soil and rock strata at Forebay, Georgetown Reservoir, and Dalecarlia WTP were estimated and presented in Table 4-2, *Summary of Soil and Rock Strength Parameters at Forebay, Georgetown Reservoir, and Dalecarlia WTP*. Competent rock is defined as bedrock that is free of weak weathering layers, or voids, or soil seams, with recovery of 80 percent and RQD of 40 percent or higher.

Table 4-1 Summary of Soil and Rock Strength Parameters at Residuals Processing Facility Site

Soil Type	General Soil Description	Unit Weight (pcf)	ϕ'^{1} (°)	c_u^{2} (psf)	q_u^{3} (psi)	q_t^{4} (psi)
Fill	Granular fill: gravel, silty sand, clayey sand	120-130	28-40 (33)[5]	-	-	-
	Cohesive fill: silt or sandy clay	110-130	-	500-2500 (900)[5]	-	-
Residual Soils	Silty sand or clayey sand	120-130	30-41 (35)[5]	-	-	-
Partially Weathered Rock	Highly weathered rock	150-160	45	-	-	-
Bedrock	Competent gneiss	160-170	-	-	2,404 – 12,979 (7,000)[5]	900-1,200 (1,000)[5]

Notes: [1] ϕ' is the drained friction angle of granular materials.
[2] c_u is the undrained shear strength of cohesive materials.
[3] q_u is the unconfined compressive strength of intact rock core samples.
[4] q_t is the split tensile strength of intact rock core samples.
[5] Average value.

Table 4-2 Summary of Soil and Rock Strength Parameters at Forebay, Georgetown Reservoir, and Dalecarlia WTP

Soil Type	General Soil Description	Unit Weight (pcf)	ϕ'^{1} (°)	c_u^{2} (psf)	q_u^{3} (psi)
Fill	Granular fill: gravel, silty sand, clayey sand	120-130	28-40 (33)[4]	-	-
	Cohesive fill: silt or sandy clay	110-130	-	500-2500 (900)[4]	-
Residual Soils	Silty sand or clayey sand	120-130	30-39 (33)[4]	-	-
Partially Weathered Rock	Highly weathered rock	150-160	45	-	-
Bedrock	Competent gneiss	160-170	-	-	4,450 – 10,100 (6,900)[4]

Notes: [1] ϕ' is the drained friction angle of granular materials.
[2] c_u is the undrained shear strength of cohesive materials.
[3] q_u is the unconfined compressive strength of intact rock core samples.
[4] Average value.

4.4 Groundwater

Based on observations during drilling, at the completion of drilling, and 24 hours or more after completion of drilling, the design groundwater elevations are presented in Table 4-3, *Design Groundwater Elevations*. These groundwater level observations were made in August and September 2006 and February and March 2007. The elevations are based on the Washington Aqueduct Vertical Datum. It should be noted that fluctuations in groundwater levels may occur due to seasonal variations, surface drainage, and other factors. Additionally, the observation results of the water table of the four temporary wells converted from borings FPS-1 at Forebay Residuals Pump Station, GPS-2 at Georgetown Residuals Pump Station, GCP-5 at Georgetown Reservoir, and BH-05 at Residuals Processing Facility Site are provided in Table 4-4, Summary of Ground *Water Table Observation Levels* . Please note the design groundwater elevations presented in Table 4-3 is an interpretation of water table observations of all borings at or near each facility. Perched water at higher elevation may be encountered during construction.

Table 4-3 Design Groundwater Elevations

Facility	**Design Groundwater, Elevation (ft)**
Residuals Processing Facility	190*
Forebay Residuals Pump Station	143
Booster Control Station at Forebay	146
Dalecarlia Residuals Pump Station	128
Georgetown Residuals Pump Station	125
Pipeline near Georgetown Reservoir	129*

*Perched water at higher elevation may be encountered during construction.

Table 4-4 Summary of Ground Water Table Observation Levels

<table>
<tr><td rowspan="2">FPS-1, with surface elevation of 156.8 ft</td><td>Observation Time</td><td>8/9/2006, before rock coring</td><td>8/9/2006, 13:10 after rock coring</td><td>8/11/2006 12:56</td><td>9/13/2006 11:00</td><td>-</td></tr>
<tr><td>Water Depth (ft)</td><td>18</td><td>15</td><td>15.3</td><td>15</td><td>-</td></tr>
<tr><td rowspan="2">GPS-2 with surface elevation of 141.9 ft</td><td>Observation Time</td><td>8/11/2006 before rock coring</td><td>8/11/2006 after rock coring</td><td>8/15/2006 8:07</td><td>8/18/2006 14:20</td><td>9/13/2006 11:45</td></tr>
<tr><td>Water Depth (ft)</td><td>27.6</td><td>20</td><td>20.4</td><td>20.3</td><td>19.5</td></tr>
<tr><td rowspan="2">GCP-5 with surface elevation of 134.7 ft</td><td>Observation Time</td><td>3/8/2007 11:30</td><td>3/9/2007</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>Water Depth (ft)</td><td>18</td><td>0*</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td rowspan="2">BH-05 with surface elevation of 219.4 ft</td><td>Observation Time</td><td>3/8/2007 15:00</td><td>3/20/2007 7:40</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>Water Depth (ft)</td><td>34.9</td><td>31</td><td>-</td><td>-</td><td>-</td></tr>
</table>

* The shallow depth of water was possibly due to melted snow on the previous and the same observation day.

4.5 Frost Penetration Depth

The frost penetration depth at the Residuals Processing Facility, Forebay, Georgetown Reservoir, and Dalecarlia WTP is 3 feet in accordance with U. S. Army Corps of Engineers (USACE) Engineer Manual EM 1110-1-1905, "*Bearing Capacity of Soils*." Therefore, the bottom of shallow foundations or column footings should be at least 3 feet below the finished grade.

4.6 Corrosivity

Corrosivity tests were performed on soil samples obtained from ten boring locations where Forebay residuals pump station, Dalecarlia residuals pump station, Georgetown reservoir pipelines, and residuals processing facility will be installed. These tests included resistivity, pH, and concentrations of chlorides and sulfides. Results from the corrosivity tests are presented in Appendix E, *Laboratory Testing Results,* and are summarized in Table 4-5, *Summary of Corrosivity Test Results*.

Based on the data presented in Table 4-5, all the soil samples tested are considered non-corrosive according to concentrations of chlorides and sulfates. However, pH measurements showed that the soil samples from GCP-3, FPS-1, BH-08, and BH-10 are acidic. Additionally, resistivity measurement of BH-08 from 33.5 to 45 feet below ground surface (bgs) indicated the soil is corrosive. All other pH and resistivity measurements indicated that the soils are not corrosive. Because corrosion of concrete is largely dependent on the concentration of sulfates, the samples tested are considered not corrosive to concrete. Also, in the range of pH 4 to pH 10, the corrosion rate of iron is relatively independent of the pH of the environment. Therefore, the samples are considered not corrosive to cast iron alloys, even though they showed acidity.

Overall, the tested soil samples from borings at Forebay residuals pump station, Dalecarlia residuals pump station, and Georgetown reservoir pipelines are considered non-corrosive according to the 10-point soil evaluation procedure by Cast Iron Pipe Research Association (CIPRA) and special protection to yard piping and foundation elements against corrosion is not necessary. At residuals processing facility site, the tested samples from shallow depth (less than 20 feet bgs) are considered to be non-corrosive and special protection to yard piping and pavement against corrosion is not necessary. However, for deep foundation design at residuals processing facility, the soil is considered to be marginally corrosive.

Table 4-5 Summary of Corrosivity Test Results

Sample ID	Depth (ft)	Soil pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
GCP-3	4 to 6	4.64	25	9	9700
GCP-5	6-10	5.27	89	13	7610
FPS-1	13.5-15	5.06	8	13	20800
PS-1	5-7	7.3	14	11	7120
BH-05	6-8	6.8	96	15	4590

Sample ID	Depth (ft)	Soil pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
BH-05	13.5-15	7.06	107	9	3090
BH-08	33.5-45	4.9	63	12	437
BH-12	18.5-25	8.09	58	14	9150
BH-13	23.5-30	6.02	51	18	8780
BH-16	13.5-20	4.1	45	13	13700

SECTION 5

Seismic Site Classification

5.1 IBC 2006 Seismic Site Class

The site class was determined following the guidelines in section 1615.1.1 of the International Building Code (2006). The steps for classifying this site were summarized below.

1. The site was determined NOT to be a Class F site because there is no liquefiable soil, quick and highly sensitive clay, collapsible weakly cemented soil, peat and/or highly organic clay, very high plasticity clay, or very thick soft/medium stiff clay.
2. The site was determined NOT to be a Class E site because the total thickness of soft clay was found to be less than 10 feet, where soft clay is defined by exhibiting undrained shear strength (c_u) less than 500 psf, water content (w) greater than 40%, and plasticity index (PI) greater than 20.
3. The site was determined to be a Class D site based on the average blow counts in the top 100 feet.

The recommended seismic design parameters for this project are summarized in Table 5-1. There parameters are developed assuming a 2% probability of exceedance in 50 years.

Table 5-1 Preliminary Seismic Design Parameters for 2 Percent Probability of Exceedance in 50 Years

Design Parameter	Value
Site Class	D
Spectral Acceleration for 0.2 sec Period, S_s (g)	0.178
Spectral Acceleration for 1.0 sec Period, S_1 (g)	0.063
Peak Ground Acceleration, PGA (g)	0.109
Site Coefficient for 0.2 sec Period, F_a	1.6
Site Coefficient for 1.0 sec Period, F_v	2.4
Magnitude of the Design Earthquake, M_w	6.4

SECTION 6

Settlement Analyses and Foundation Recommendations

6.1 Shallow Foundations and Settlement Analyses

The net allowable bearing capacity of soils for shallow foundations is estimated based on safety factor of 3 and using the methods provided in EM 1110-1-1905, "*Bearing Capacity of Soils*.", The net allowable bearing capacity and the estimated minimum embedment depth are presented in Table 6-1, *Estimated Net Allowable Bearing Capacity*. Additionally, the estimated gross and net applied bearing pressure at the bottom of foundations and foundation size are presented in Table 6-1. The bearing capacity and settlement analyses were not performed for Dalecarlia Residuals Pump Station because it was determined to support this structure on deep foundation due to the presence of uncontrolled fill and an existing 36-inch reinforced concrete (R. C.) Drain underneath the footprint of this structure. Due to the random nature of fill observed in the boring, the settlement analysis based on one boring may not be representative. Any settlement as a result of the installation of the pump station may damage the existing utility below it. Therefore, it is recommended to support Dalecarlia Residuals Pump Station on deep foundations.

Table 6-1 Estimated Net Allowable Bearing Capacity

Structure	Minimum Embedment (ft)	Net Allowable Bearing Capacity (psf)	Footing Width (ft)	Footing Length in One Side of Structure (ft)	Gross Applied Pressure (psf)	Net Applied Pressure (psf)
Residuals Processing Facility – Columns	3	3,000	18 to 19	18 to 19	3,360	3,000
Residuals Processing Facility – Basement Wall	16	6,000	5 to 9	40 to 90	8,180	6,000
Residuals Processing Facility – Gravity Thickeners	5	3,000	-	-	3500	2,950
Forebay Residuals Pump Station	19	6,000	19	43	1,480	0
Booster Control Station	3	3,000	3	15	1,000	670
Georgetown Residuals Pump Station	3	1,500	23	55	1000	670

The settlement potential s of various structures in this project, were estimated using Schmertmann's strain influence method (Schmertmann, 1970; Schmertmann, et al. 1978) and Burland and Burbidge's method (1985), provided in EM 1110-1-1904, "*Settlement Analysis*" to determine whether shallow foundations are suitable or not.

Based on the applied foundation pressure and foundation size in Table 6-1, the potential settlements of various structures were estimated and presented in Table 6-2, *Summary of Potential Settlements and Foundation Recommendations*, if shallow foundations are used. The settlement due to the fill materials outside of the gravity thickener tanks was included in addition to that from the structure loading of the gravity thickener tanks. The allowable total and differential settlement are recommended to be 1 and ½ inch, respectively, according to EM 1110-1-1904 for most buildings. Based on these criteria and potential settlements, the foundation recommendations of supporting various structures on shallow or deep foundations are summarized in Table 6-2.

Table 6-2 Summary of Potential Settlements and Foundation Recommendations

Structure	Minimum Total Settlement of Shallow Foundation (in.)	Maximum Total Settlement of Shallow Foundation (in.)	Differential Settlement of Shallow Foundation (in.)	Recommendation to Support the Structure on Shallow or Deep Foundations	Modulus of Subgrade Reaction for Shallow Foundations, K_{V1} (tons/ft^3)
Residuals Processing Facility – Column Footing	1	5	0.5-1.2	Deep Foundations	N/A
Residuals Processing Facility – Wall Footing	0.2	4	2.5-3.5	Deep Foundations	N/A
Residuals Processing Facility – Gravity Thickeners	0.6	4	2-3	Deep Foundations	N/A
Forebay Residuals Pump Station	0.1	0.1	0.1	Shallow Foundations	150
Booster Control Station	0.2	0.4	0.1-0.3	Shallow Foundations	30
Georgetown Residuals Pump Station	0.1	0.2	0.1-0.2	Shallow Foundations	45

As shown in Table 6-2, the use of shallow foundations is acceptable for the Forebay Residuals Pump Station, Booster Control Station, and Georgetown Residuals Pump Station. However, shallow foundations are not suitable for the Residuals Processing Facility and

Dalecarlia Residuals Pump Station. For the Residuals Processing Facility, it is recommended to use deep foundations to support the structures because of probable excessive settlement and probable large different settlement due to the existing uncontrolled dredged fill in this area. The type of deep foundations to be used at the Residuals Processing Facility is discussed in the next section.

6.2 Deep Foundations for the Residuals Processing Facility and Dalecarlia Residuals Pump Station

6.2.1 Foundation Type

Several types of deep foundations, such as driven piles, augered cast-in-place (ACIP) piles, drilled shafts, and micropiles were considered in the design. The presence of boulders (up to 13 feet thick) as indicated in the recent boring logs (WA-1 to WA-4) and phase II borings (BH-01 to BH-16), suggests that installation of driven piles and ACIP piles will be very difficult if not impossible. Drilled shafts were considered in concept design stage. However, Phase II subsurface investigation showed that the extent and size of boulders (up to 13 feet or larger) and other obstructions, including concrete, brick, slag, and asphalt will pose significant challenges to the drilled shaft construction. Per discussion with three qualified drilled shaft contractors, the construction time and cost may be double or triple what is typically needed due to the presence of large size boulders and other obstructions. Additionally, it is difficult to relocate the location of drilled shafts since the size of boulder may be more than 13 feet.. On the other hand, micropiles can be constructed through boulders relatively easier due to the small diameter which can maneuver or core through boulders. The cost of micropiles is a bit cheaper than drilled shafts at this site due to presence of boulders. Another advantage of the micropile is that the cost can be estimated per unit length regardless of the type of materials being drilled through. Therefore, it is recommended to support residuals processing facility using micropiles socketed in bedrock.

At Dalecarlia residuals pump station, extensive existing utilities are located adjacent and underneath the proposed footprint of the pump station. It is preferred to use smaller diameter deep foundation to minimize potential damage to existing utilities. Therefore, micropiles are recommended to support Dalecarlia residuals pump station.

6.2.2 Micropile Design

Micropile design was based on Federal Highway Administration (FHWA) Report Micropile Design and Construction Guidelines, Publication No. FHWA-SA-97-070 by Armour, et al. (2000).

The maximum design load of the columns at residuals processing facility is 1400 kips. It was decided to use four micropiles with design capacity of 350 kips to support each column. Micropiles with design capacity of 350 kips were also used to support the Dalecarlia residuals pump station and the basements and gravity tanks of the residuals processing facility except the locations subjected to downdrag load.

For geotechnical design, the ultimate bond strength between competent rock (gneiss) and grout was assumed to be 200 psi according to Armour et al. (2000). The resistance from soil in unbonded zone is ignored. A safety factor of 2.5 was used for rock-grout bond strength. To achieve 350 kips of design capacity, 8-inch diameter micropiles socketed in competent rock 18 feet is recommended.

For structural design of the micropiles, the equation $0.33*f'_c*A_c + 0.4*f_y*A_s$ (where f'_c is compressive strength of grout, A_c is cross section area of grout, f_y is yield strength of reinforcement, A_s is cross section area of reinforcement) for estimating structural capacity of micropile instead of $0.4*f'_c*A_c + 0.47*f_y*A_s$ provided by Armour et al. (2000) was used in the design per conservations with micropile specialty contractors. The conservative equation used herein is to be adopted in an upcoming FHWA report. In order to achieve 350 kips of design structural capacity, API N-80 casing with 7-inch outer diameter and 0.5-inch wall thickness was selected as permanent casing. Additionally, one #14 rebar with a minimum 75 ksi yield strength within the casing is required to resist 350 kips vertical design load. The plunge length of the casing was estimated to be 14 feet. The residuals processing site was considered to be non-corrosive to marginally corrosive. For micropile design, the site was conservatively considered to be marginal corrosive. A 1/16 inch reduction in the casing thickness was considered per Armour et al. (2000).

For three out of the four gravity tanks, additional fill up to 7-foot thick will be placed surrounding the tanks, which will cause downdrag loads on the micropiles supporting the tank walls. A downdrag load of 137 kips was estimated on each micropiles along the perimeter of the tanks and near the fill. To accommodate the downdrag load, the design capacity of 300 kips was used for the micropiles supporting gravity tank walls and near the fill. The same micropile design for 350 kips design capacity, except that the center rebar is one #20 bar with a minimum 80 ksi yield strength, was designed for the 300 kips design load and 137 kips downdrag load.

The vertical settlement and lateral deflection of micropiles were estimated using FB-Multipier by Bridge Software Institute (BSI) and LPILE Plus 5.0 by Ensoft, Inc. Due to the small diameter, the settlement of micropile is mainly due to elastic deformation of the micropile elements. The maximum settlement under 350 kips design load was estimated to be 0.5 inches using the longest micropile, which was based on the lowest bedrock elevation observed from the borings. The lateral deflection of micropiles under seismic induced shear force was estimated to be less than 0.5 inches.

The basement walls along axis 3 and 8 will be temporary loaded by the earth pressure from the soil behind the wall during construction. One row of micropiles battered 14 degree from vertical (1 H to 4 V) and another row of vertical micropiles spaced at 5 feet interval were designed to support the two walls. It was assumed that the walls behave as a cantilever wall under the temporary condition when the ground floor slabs and structures above it are not yet constructed. The loads acting on the walls at this time are the active lateral earth pressure and surcharge load.

Computer program FB-Multipier by BSI was used to estimate the lateral deflection of the walls. The thickness of the wall used in the analysis was 2 feet. A surcharge pressure of 250 psf was considered in the lateral analysis under the temporary condition. As a result, heavy construction equipment should be kept a distance of 15 feet away from these walls and the backfill behind these walls should not be compacted with a heavy roller. The steel reinforcement consists of #6 bars at 6-inch spacing in both sides of the walls was modeled in the FB-Multipier analyses. The lateral deflection at the top of basement walls was estimated to be less than 0.5 inches.

Based on the above design results, the design of micropiles for various structural elements of the Residuals Processing Facility and Dalecarlia residuals pump station are summarized in Tables 6-3, *Summary of Micropile Design*. The final micropile tip elevations depended upon the elevation of bedrock surface and verification load test results of micropiles.

Table 6-3 Summary of Micropile Design

Location	Design Load (kips)	Downdrag Load (kips)	Approximate Micropile Top Elevation (ft)	Estimated Elevation of Rock Surface (ft)	Estimated Elevation of Micropile Tip (ft)	Estimated Length of Micropile (ft)	Batter Angle from Vertical (degree)
Residual Building	350	0	214	147	129	85	0
Battered Micropile for the Walls along Axis 3 & 8	350	0	199	149	131	68	14
Basement	350	0	199	150	132	67	0
Micropiles under Gravity Wall and near Fill	300	137	211	156	138	73	0
Gravity Tank	350	0	202 to 211	157	139	63-72	0
Dalecarlia Residuals Pump Station	350	0	130	108	90	40	0

Note:
1. Rock socket length was estimated to be 18 feet. Micropile tip elevations provided in above table were estimated based on rock surface elevation from soil borings. The final rock socket length will be provided upon completion of static load test program for the micropiles.
2. Factor of safety of grout-rock bond strength is 2.5 for geotechnical capacity design.
3. Ultimate geotechnical capacity shall be calculated as Factor of Safety * Design Load + Downdrag Load.
4. The total load for structural design was Design Load + Downdrag Load.

6.3 Deep Foundations for Winches

Winches and static tension cables are proposed to be installed in the embankment slope of the Georgetown Reservoir and Forebay to dredge the reservoirs. Deep foundations, including driven piles, micropiles, and drilled shafts, were considered to support the winches due to large lateral load from the static cables. Due to the relatively shallow depth of rock approximately a few feet to 20 feet below the bottom of reservoir, driven piles will have inadequate embedment length; therefore, driven piles were not selected. A group of battered micropiles may provide adequate lateral resistance. However, a construction platform or a barge will be required to install a group of battered micropiles battered in different directions. The cost of a construction platform hanging above the reservoirs or a barge will be very expensive. Additionally, the construction platform itself may disturb the operation of the reservoir. Based on above considerations, large diameter drilled shafts are recommended to support the winches and resist the lateral forces applied from the cables.

The design load combinations at the elevation of the static cables for load cases 1, 2, and 3 and at the top of drilled shaft for load case 4 are provided in Table 6-4, Winch Foundation Design Loads. The elevation of the drilled shaft will be 3 feet to 10 feet below the elevation of the cables.

Table 6-4 Winch Foundation Design Loads

Load Case	Axial Compression (kips)	Lateral Load (kips)	Torsion (kip-ft)
1	0	92	46
2	0	86	94
3	0	86	75
4	30	0	0

Borings GCP-2, GCP-4, GCP-6, GPS-3, and GPS-4 were considered to develop a design soil profile for drilled shafts supporting the winches at Georgetown Reservoir. At the Forebay, borings FPS-1 and FB1 were used to develop a design soil profile. Some soil resistance on the slope was conservatively ignored during analysis.

One single 5-foot diameter drilled shaft was designed to support each winch with 92 kips or lower lateral load from the cables. The shaft diameter was determined using computer program FB-Multipier v 4.08 by BSI, Inc. The maximum estimated lateral deflection of at the elevation of the tension cable was estimated to be 1.6 inches, which is considered acceptable since the cable length can be adjusted. Static p-y curves build in the program were used. The length of drilled shaft was determined by ensuring adequate fixity in the tip of drilled shaft for resisting lateral loads.

For winch slabs without lateral load (i.e. load case 4), 3-ft diameter drilled shafts were designed to resist 30 kips axial compression load. Factor of safety of 3 was used. Computer program SHAFT by Ensoft, Inc. was used to estimate axial capacity and axial settlement.

Based on above analyses, the design of drilled shafts for winches at Georgetown Reservoir and Forebay are summarized in Tables 6-5, *Summary of Drilled Shaft Design*. The final drilled shaft tip elevations depended on the elevation of bedrock surface.

Table 6-5 Summary of Drilled Shaft Design

Locations	Diameter of Drilled Shaft (ft)	Rock Socket Length (ft)	Permanent Casing Thickness (in)	Approximate Drilled Shaft Top Elevation (ft)	Estimated Potential Lowest Drilled Shaft Tip Elevation (ft)
Georgetown Reservoir – 5-ft Diameter Drilled Shafts	5	5 ft or less	0.25	145 to 150	99
Georgetown Reservoir – 3-ft Diameter Drilled Shafts	3	0	0.25	149	129
Forebay – Drilled Shafts	5	8 ft or less	0.25	150	104

Note:

1. For 5-foot diameter drilled shaft at Georgetown Reservoir, the drilled shaft shall be socketed into competent rock 5 ft or with a tip elevation of 99 feet, which ever is shallower.
2. At Forebay, the drilled shaft shall be socketed into bedrock 8 ft or with a tip elevation of 104 feet, which ever is shallower.
3. Permanent casing shall be used and shall have minimum yield strength of 35 ksi. The permanent casing shall extend to the bottom of the drilled hole or top of bedrock. The diameter of rock socket shall be 6” less than the diameter of the drilled shaft in soils.

SECTION 7

Excavation Support Recommendations

7.1 Lateral Earth Pressure

Recommended design equivalent fluid lateral earth pressures, for either temporary or permanent structures, imposed by fill and native soils, are presented in Table 7-1, *Lateral Earth Pressure*.

Table 7-1 Lateral Earth Pressure

Condition	Equivalent Fluid Pressure Above the Groundwater Table (psf/foot of height)	Earth Pressure Below the Groundwater Table (psf/foot of height)
Active	45	85
At-rest	65	95
Passive	375	250

Surcharge loads from temporary construction equipment or permanent structures should be added to the lateral earth pressure with an active earth pressure coefficient of 0.35 and at-rest earth pressure coefficient of 0.5. Surcharge load from temporary construction equipment should be equivalent to 650 psf applied at the ground surface.

7.2 Types of Excavation Support Systems for Major Structures

The excavation support systems required to construct the various structures/facilities are summarized in Table 7-2, *Recommended Excavation Support Systems for Major Structures*. The approximate bottom elevations of excavation, design groundwater elevations, recommended types of excavation support systems, and considerations to recommend the excavation support systems are also presented in Table 7-2.

It is noted in Table 7-2 that the design groundwater elevation is above the bottom of excavation at the Forebay Residuals Pump Station location. The recommended soldier pile and lagging wall for excavation support is not a groundwater cutoff wall. As a result, it is necessary to dewater using dewartering wells before excavation starts. For Forebay residuals pump station and Dalecarlia residuals pump station, the soldier pile and lagging wall are recommended to service as forms for construction of the pump station walls. These excavation supports will be left in place. However, they should be demolished to minimum 3 feet below the bottom of the proposed pipelines.

The excavation support at Georgetown residuals pump station may be removed to allow for the installation of pipes. For all other areas, the excavation support may be left in place but cut the top portion of piles 3 to 5 feet below the finished ground surfaces.

Table 7-2 Recommended Excavation Support Systems for Major Structures

Structure	Approximate Bottom Elevation of Excavation (feet)	Maximum Exposed Excavation Depth (feet)	Design Groundwater Elevation (feet)	Recommended Type of Excavation Support System	Considerations to Recommend the Excavation Support System
Forebay Residuals Pump Station	136.5	20	143	Drilled-in Soldier Pile and Lagging Wall with 2 Levels of Lateral Bracing (Soldier Piles Spaced at 8 to 10 feet)	There is an existing 9-ft diameter brick conduit near the pump station. An excavation support system is required to construct the pump station while protecting the brick conduit. The approximate depth of excavation is 19 feet. The top of rock is approximately at an elevation of 133.5 feet. Driven sheet pile will cause vibrations that can damage the brick conduit and will be refused on the top of rock. There will not be sufficient embedment to develop lateral resistance for the sheet pile wall. Therefore, a drilled-in soldier pile and lagging wall with 2 levels of lateral bracing is recommended. Due to site constraints, internal bracing may be installed at the top of the wall and rock anchor may be installed 5 feet above the bottom of the excavation.
Dalecarlia WTP Yard Piping	varies	12	128	Trench Box	The bottom of excavation of various yard piping varied from 6 to 12 feet below the existing grade. There are existing duct bank and 12-in. diameter RC drains encased in concrete running parallel and within 5 feet from the excavation. Additionally, a 24-inch water main is within 5 feet away from a proposed 8” pipe. Due to the site constraints, trench boxes are recommended to install the yard piping. The outside width of the trench box should not less than the width of excavation minus 6-inch.

Dalecarlia Residuals Pump Station	131	15	128	Drilled-in Soldier Pile and Lagging Wall (Soldier Piles Spaced at 8 to 12 feet)	There is an existing 36-in. diameter RC drain encased in concrete running under the footprint of the pump station and other proposed yard piping in the vicinity. An excavation support system is required to construct the pump station while protecting the existing utilities. Driven sheet pile will have physical conflicts with the RC drain. Additionally, vibrations caused while driving the sheet pile can damage the existing utilities and nearby sedimentation basins. The approximate depth of excavation is 15 feet. Because the pump station is bounded by two sedimentation basins, an L-shaped excavation support system is recommended. Internal bracing above the construction joints of the pump station wall can be installed by utilizing the support from the sedimentation basin walls. Therefore, a drilled-in soldier pile and lagging wall with one level of internal bracing is recommended. The spacing of the solder piles is recommended to be between 8 and 12 feet to limit the lateral deformation to an acceptable value and avoid to damage existing utilities.
Georgetown Residuals Pump Station	128.5	19	125	Driven Sheet Pile Wall (Fitted with Cast Steel Protectors at the Tip) with Deadman Anchors	The excavation is between 30 and 50 feet from the edge of the gravel road around Georgetown reservoir. The maximum depth of excavation is 19 feet. Georgetown reservoir embankment is considered a levee. Open cut in this embankment is not recommended. Therefore, an excavation support system is required to construct the pump station. Existing grade is sloping downward toward to northwest at a slope ratio between 3H:1V and 4H:1V. A sheet pile wall with deadman anchors installed on the east and south sides of the excavation is recommended. Due to potential hard driving conditions expected near the end of installation, sheet piles should be fitted with cast steel protectors to facilitate installation.
Pipeline Near Georgetown Reservoir	131	5	129	Trench Box	The trench excavation is more than 15 feet away from the toe, therefore, trench box is recommended as the excavation support system. See the discussion in Section 7.3.

7.3 Slope Stability Analyses of Trench Excavation for Pipelines

The Georgetown Reservoir embankments are classified as levees. Slope stability analyses of the existing reservoir embankment slope was performed to study its stability and to evaluate the stability of a trench excavation near the toe of the embankment to install a pipeline.

The geometry of the slope was established based on the topographic map from a recent survey and limited historical drawings. Soil profiles in the cross-section for slope stability analyses were developed from two borings, GCP-4 and GCP-3, drilled at the crest and toe of the embankment, respectively. Borings GCP-1, GCP-2, GCP-5 and GCP-6 were also used to develop design soil parameters. The shear strength parameters of soils, as summarized in Table 7-3, *Summary of the Soil Parameters Used in the Slope Stability Analyses*, were developed based on correlations with SPT N-values and laboratory consolidated-undrained (CU) triaxial tests with pore pressure measurements, performed on a relatively “undisturbed” clayey soil sample obtained from offset borehole of GCP-6 at the crest of the slope. Other soil parameters, such as the unit weight and hydraulic conductivity, were developed based on soil classifications and the relative density as indicated by the SPT N-value. These parameters are also presented in Table 7-3.

Table 7-3 Summary of the Soil Parameters Used in the Slope Stability Analyses

Soil Type	Unit Weight, γ (pcf)	Effective-stress Cohesion, c’ (psf)	Effective-stress Friction Angle, ϕ’ (deg.)	Total-stress Cohesion, c_u (psf)	Total-stress Friction Angle, ϕ_u (deg.)	Hydraulic Conductivity, k (m/sec)
Granular Fill	120	0	31	0	31	10^{-6}
Cohesive Fill and Native Clayey Material	130	0 or 102*	31	1000	0	10^{-8}
Residual Soil (Silty Sand)	140	0	45	0	45	10^{-6}

* The effective stress cohesion of 102 and 200 psf were measured from two “undisturbed” soil sample obtained from an offset boring of GCP-6 and a nearby boring GPS-2 using consolidated undrained Triaxial test with pore water pressure measurement. However, zero effective cohesion is also considered in analysis as worst case scenario.

Combined seepage and slope stability analyses were performed using a computer program, SLIDE version 5.026, developed by Rocscience, Inc. Long-term conditions, using effective-stress shear strength parameters, were assumed in the analyses of the stability of the existing embankment. The high water level in the reservoir elevation of 148 feet was assumed for the analyses. Steady-state seepage analyses were performed to establish the groundwater distribution within the embankment prior to performing the slope stability analyses. The factors of safety of the stability of the existing embankment slope, according to the simplified Bishop method, are 1.8 and 1.3, corresponding to effective cohesion of 102 psf and 0 psf of the cohesive soil layer, respectively. The actual factor of safety may be between 1.3 and 1.8 considering non zero cohesion were measured from both consolidated undrained Triaxial test with porewater measurement. These analyses validate that the existing embankment slope is stable, as observed in the field. These analyses validate that the existing embankment slope is stable, as observed in the field.

Further, slope stability analyses were performed to evaluate the impact of trench excavation near the toe of this embankment slope on its stability. The dimensions of the trench were assumed to be 4-ft wide and 5-ft deep at the maximum. Two possible locations of the trench were considered. The first location is between the toe of the slope and the inner fence. The second location is between the inner and outer fences of the reservoir. Centerlines of the trenches were assumed to be at an offset distance of 6 and 21 feet from the toe of the slope. The trenches were assumed to be cut with vertical slope due to the limits of space for sloped cut. In these analyses the water level in the reservoir was assumed to be at an elevation of 148 feet.

Both short-term and long-term conditions, using the total-stress and effective-stress shear strength parameters, were considered. The embankment factors of safety from short-term stability analyses of the two trench options are1.3, which correspond to a potential failure mode in the upper embankment. For both proposed trenches indicated above, the potential short-term failure modes with failure surface passing through the top of embankment and the proposed trenches have a factor of safety greater than 2.8. This indicates that both the trench itself with vertical slope and embankment slope will be stable under the short-term conditions.

For long-term stability analysis, worst effective soil parameter, i.e. zero effective cohesion, was used. The long-term factor of safety of the embankment through the potential trench 6 feet and 21 feet away from the toe are 1.2 and 2.4, respectively. However, the lowest factors of safety for the two trench options are less than 0.3, which correspond to a local trench failure mode. Considering the time involved to install the pipeline, long-term conditions may be relevant if the trench is left open for a few weeks. It is important to note that the factor of safety of 0.3 is for the trench itself only and is not the direct indication of the stability of the existing embankment. However, the local failure of trench may detriment the stability of the embankment. Therefore, an excavation support system is recommended to install the pipeline.

Trench box may not be an adequate excavation support system for the first location between the toe of the slope and the inner fence because when the soil outside the trench box fails, it usually results in significant movement that can cause further progressive failures of upper soil masses in the embankment. A continuous sheet pile wall is the most appropriate excavation support system to ensure the stability of levee for the first trench location.

On the other hand, trench box will be adequate for the second trench location between the inner and outer fences. Because soil movement near the trench should not result in further weakening of soil in the embankment. Therefore, trench box is recommended for the installation of the pipe since it will be located approximately 20 feet away from the toe of the slope.

SECTION 8

Pavement Design

8.1 Site Road at the Residuals Processing Facility

Based on client provided information, the normal operation hours of the Residuals Processing Facility will be from 7 A.M. to 7 P.M. Trucks of 3 axles with a gross weight up to 80 kips (40 tons) will come to and leave the Residuals Processing Facility at a frequency of 7 to 25 times a day and 5 days a week. A small parking lot of 5 stalls is planned at south of the facility near the southeast gravity thickener. Both flexible and rigid pavements were initially evaluated during the preliminary design for the site roadway at Dalecarlia Residuals Processing Facility. The reinforced rigid pavement is recommended in the final design because heavy vehicles dominate the mixed design traffic during the 20-year design life. A Pavement-Transportation Computer Assisted Structural Engineering (PCASE) software is used to perform the thickness design of reinforced concrete pavements. The PCASE version 2.08 software conforms to USACE Manual TM 5-822-5, "Pavement Design for Roads, Streets, Walks, and Open Storage Areas." The USACE Manual TM 5-822-5 is also used to design reinforcing steels, joints and joint sealing.

8.1.1 Traffic Information

Numbers of trucks and passenger vehicles for a 20-year design life are assumed below. Since the maximum anticipated daily truck traffic (25 trips) is used to estimate the 20-year design traffic, a growth factor is excluded in projecting the design traffic. The maximum axle weights for the front, middle and rear axles of design trucks are 12 kips, 34 kips, and 34 kips, individually.

$$[(\text{PASSES}_{\text{Passenger Car}})_{\text{One-Way}}]_{\text{20-Year}} = (\text{5-car} \times \text{3-trip}) \times \text{25-day} \times \text{12-month} \times \text{20-year}$$

$$= 90{,}000 \text{ passes}$$

$$[(\text{PASSES}_{\text{Truck}})_{\text{One-Way}}]_{\text{20-Year}} = (\text{25-trip}) \times \text{25-day} \times \text{12-month} \times \text{20-year}$$

$$= 150{,}000 \text{ passes}$$

Using the PCASE computer program, a critical vehicle (i.e. the 3-axle truck) is selected and the equivalent passes (150,001) of the critical vehicle for the mixed traffic are obtained as shown in Table 8-1. Per USACE TM 5-822-2, the site roadway is a class F road assuming flat terrain. The traffic category is IVA per USACE TM 5-822-5 and the pavement design index is 4.

Table 8-1 Traffic Inputs of Pavement Design at Residuals Processing Facility

Vehicle	Total Weight (lb)	Individual Passes	Equivalent Passes Using Critical Vehicle
Car-Passenger	4,000	90,000	1
Truck, 3 axle	80,000	150,000	150,000
Truck, 3 axle	80,000		150,001

8.1.2 Rigid Pavement Design

A joint reinforced concrete pavement (JRCP) is designed for the site roadway at the Residuals Processing Facility to accommodate heavy truck traffic.

8.1.2.1 Modulus of Subgrade Reaction

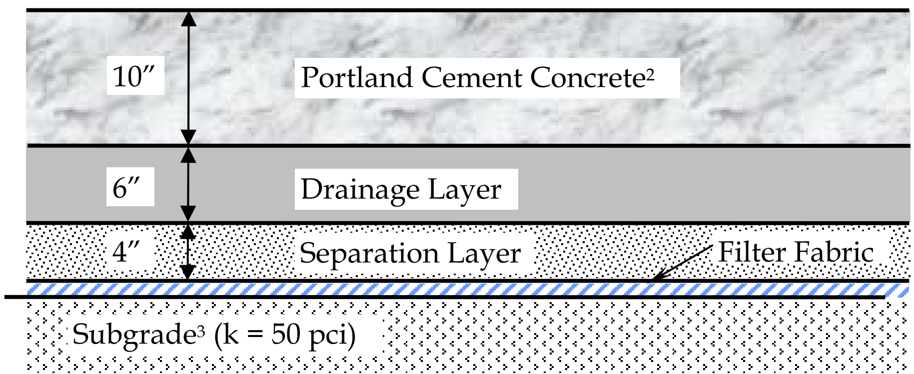
According to boring logs, sandy silt and silty sand are present below the finished grade. In addition, results of laboratory California Bearing Ratio (CBR) tests show an average CBR value of 2.9 at 95 percent compaction. Since a field plate-loading test is not planned, a modulus of subgrade reaction (k-value) is assumed based on soil classification and properties. Per USACE TM 5-822-5, for the silts and clays soil groups with liquid limits less than 50 and moisture content over 28%, an estimated k-value is 50 pci. To consider subsurface variations and seasonal moisture changes, it is recommended to prepare the subgrade of k-value of 100 pci or higher prior to rigid pavement construction. For a conservative design, a subgrade k-value of 50 pci is assumed in the rigid pavement thickness design.

8.1.2.2 Thickness of Strength Design for Rigid Pavement

Based on the precipitation database in PCASE software, using the Washington DC Westbound City Station in Maryland, would indicate a maximum daily precipitation of 2.4 inches for a 2-year design period. The length of drainage path is 26.83 feet as calculated by PCASE assuming the drainage layer has a 12-feet length of 2% transverse slope, and a 4% longitudinal slope. Uniformly graded aggregates are used in the drainage layer and with a permeability of 2,500 feet per day. The effective porosity and infiltration coefficient are assumed to be 0.3 and 0.5, respectively. Based on the above input parameters, the minimum required thickness of the drainage layer is 4.8 inches. A thickness of 6-inch drainage layer using rapid draining material with a permeability of 2,500 feet per day or greater is recommended. A granular separation layer of 4-inch is needed to prevent fines from infiltrating into the drainage layer. A filter fabric (geotextile) is also recommended to be placed directly on subgrade to provide extra prevention of pumping of fines into the overlaying layer. Material strengths, gradations and properties of drainage and separation layers as well as properties of filter fabric should meet requirements set forth in USACE Manual EI 02C202, “Subsurface Drainage for Pavements.”

Using a concrete 28-day flexural strength of 650 psi, a subgrade k-value of 50 pci, a drainage base of 6-inch and a separation subbase of 4-inch, the minimum required thickness of concrete slabs is 8-inch assuming at least 25% load transfer is achieved during the design period. An added 2-inch of concrete slabs would be required if no dowels were used for load transfer across joints. Since heavy truck traffic dominates the mixed traffic and a substandard load transfer may be encountered towards the end of the design life, concrete slabs of 10-inch rather than the minimum required 8-inch are recommended. Based on the strength design criteria, the recommended thickness of a rigid pavement at the Residual Processing Facility site consists of 10-inch concrete slabs, a 6-inch drainage base layer, a 4-inch separation layer, and a filter fabric (geotextile) over the subgrade as shown in Figure 8-1.

Figure 8-1 The Recommended Thickness Design[1] of Rigid Pavement



[1] Not drawn to scale
[2] Modulus of elasticity of concrete (E_{PCC}) = 4,000,000 psi;
ultimate compressive strength (f'_c) = 4,000 psi;
steel reinforcements not shown
[3] To account for subgrade strength reduction due to seasonal moisture changes, it is recommended to provide a prepared subgrade of k = 100 pci or higher prior to construction

8.1.2.3 Check Strength Design Thickness of Rigid Pavement against Frost Protection

Per USACE TM 5-822-5, the frost groups of Residual Processing Facility soils are F3 and F4. Using the depth of frost penetration calculator in PCASE, minimum total thicknesses of frost design using two methods (i.e. limited subgrade frost penetration method and reduced subgrade strength method) are shown in Table 8-2. Due to a low design freezing index at the Dalecarlia Reservoir Weather Station, the frost depth design of the rigid pavement is governed by the method of limited subgrade frost penetration because a lesser (or a more economical) thickness is required.

Assuming nonfrost-susceptible materials are used in paving, the recommended rigid pavement thickness based on strength design criteria as shown in Figure 8-1 is able to prevent frost penetration into the frost-susceptible subgrade.

Table 8-2 Minimum Thicknesses of Frost and Strength Design of Rigid Pavements

Min. Thickness (in) / Layer	Reduced Subgrade Strength Method[6]	Limited Subgrade Frost Penetration Method[5,6]	Strength Design (Non-Frost Design)
Concrete Slabs[4]	9	8	8
Drainage	5	5	5
Separation	4	4	4

[4] At least 20% load transfer is achieved
[5] Depth of frost = 13” (calculated using PCASE)
[6] The design air freezing index = 279.4 degree-days and mean annual temperature = 55.9 °F (for Dalecarlia Reservoir Weather Station in Maryland)

8.1.2.4 Reinforcing Steels and Load Transfer

Number 8 dowels, 18 inches in length and 12 inches on center, are recommended at transverse joints to provide load transfer across joints under heavy, repeated truck traffic. Number 5 tie bars, 36 inches in length and 30 inches on center, are also recommended at longitudinal construction joints to ensure good joint load transfer.

To hold slabs or cracks tightly after cracking, to reduce differential settlements, and to prevent further deteriorations (i.e. faulting and pumping) coming from cracks or joints, reinforcing bars are recommended for the rigid pavement. Number 5 reinforcing bars spaced longitudinally at 8 inches on center and transversely at 12 inches on center are recommended. To provide a durable joint sealing and to reduce future maintenance costs, preformed compression sealants are recommended for this JRCP. The maximum recommended transverse joint spacing is 40 feet that was found to be the most economical joint spacing (Nussbaum and Lokken, 1978).

8.2 Gravel Roads at Georgetown Reservoir Pump Station and Dalecarlia Residual Processing Facility

USACE TM 5-822-12, Design of Aggregate Surfaced Roads and Airfields, is used to perform thickness design of gravel roads at Georgetown Reservoir Pump Station and Dalecarlia Residual Processing Facility.

8.2.1 Traffic Information

Since very limited traffic of maintenance vehicles is expected at two project sites, the lightest road class, class G, is assumed for gravel roads in the design. The traffic category is assumed to be type IVA that includes three-axle trucks. Per USACE TM 5-822-12, the design index is 2.

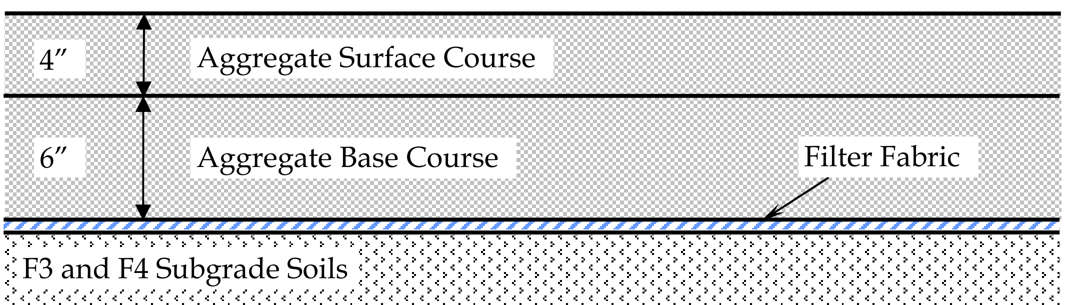
8.2.2 Soil Support Index of Subgrade

According to boring logs at Georgetown Pump Station, silty gravel and clayey gravel are present in the subgrade. Per USACE TM 5-822-12, the frost group of subgrade soils at Georgetown Pump Station is F3. Base on boring logs of Dalecarlia Residual Processing Facility, sandy silt and silty sand are present in the subgrade. Per USACE TM 5-822-12, the frost group of subgrade soils at Residual Processing Facility is F4. Based on USACE TM 5-822-12, a soil support index of 3.5 is used for F3 and F4 subgrade in pavement thickness design to account for frost depth design.

8.2.3 Thickness of Gravel Roads

According to USACE TM 5-822-12, a minimum of 8-inch thickness is required for gravel roads. To consider aggregate losses for a 20-year design life, an additional 2-inch base course is provided. The recommended thickness of gravel roads for two project sites consists of 4-inch aggregate surface course, 6-inch aggregate base course, and a filter fabric (geotextile) over the subgrade as shown in Figure 8-2. A filter fabric is recommended due to the presence of F3 and F4 subgrade soils.

Figure 8-2 The Recommended Thickness Design of Gravel Roads



Note: Not drawn to scale

SECTION 9

Construction Considerations

9.1 Fill and Backfill

Site fill is expected to be placed at the Residuals Processing Facility site. The major sources of fill or backfill will be on-site. Imported material may be used when satisfied on-site material is inadequate.

The fill and backfill materials should meet the following requirements:

- Maximum of 35 percent passing the No. 200 sieve, maximum liquid limit of 40, maximum plasticity index of 20.
- Free of roots, debris, organic material, stumps and limbs, manmade waste, or any other unsuitable material that would create a potential hazard during any future excavation.
- Maximum particle size not exceeding 1.5 inch. However, for common site fill away from structures and pavement or under pile supported structures, the maximum allowable particle size of processed rock can be 3 inches.

For the compaction of the fill and backfill, the following guideline is recommended:

- Uniformly moisten or aerate subgrade and each subsequent fill or backfill soil layer before compaction to within 2 percent of optimum moisture content.
- Compact maximum 6-inch loose lift soil materials to not less than the following percentage of maximum dry density according to ASTM D1557 (Method C):
  - Under the Georgetown residuals pump station, Forebay residuals pump station, Booster Control Station, scarify and re-compact top 12 inches of existing subgrade and each layer of backfill or fill soil material at 95 percent.
  - Under micropile supported structures, including residual processing facility and Dalecarlia residuals pump station, compact each layer of backfill or fill soil material at 90 percent.
  - 5-foot zone adjacent to structure walls, compact each layer of backfill or fill soil material at 90 percent.
  - Behind the basement walls along axis 3 and 8 of the residuals processing facility, the compaction effort should be minimum. The edge of compactor should at least 2 feet away from the edge of the wall.
  - Under the pavement in residuals processing facility site, scarify and re-compact top 12 inches of existing subgrade and each layer of backfill or fill soil material at 97 percent.

9.2 Excavation and Excavation Slopes

For soils with SPT N-values less than 100 bpf, conventional earth-moving equipment, such as backhoes or bull dozers with rippers, and front-end loaders, can be used for excavation of soil and highly weathered rock. Partially weathered rock with SPT N-values less than 100 blows-per-4-inches may be rippable. However, hydraulic rams or other suitable mechanical techniques of the contractor's choosing will be necessary to excavate large size boulders. No blasting is permitted.

For the Booster Control Station at Forebay, the loose sand of black color encountered in boring FB-1 is recommended to be excavated and replaced with granular fill to a depth of 1 feet below bottom of foundation or dense soil is encountered if it is encountered during excavation. The bottom of Georgetown Residuals Pump Station is stepped. It is recommended to excavate within the entire footprint of the pump station to the lowest excavation level to remove soft clay that is potentially present and backfill with granular fill to the foundation levels.

Based on the subsurface conditions, using temporary excavation slope of 2H:1V is recommended. Excavations in the weathered rock and boulder zone may be cut more steeply, although provisions to control raveling of loose rock and soil chunks are required.

9.3 Trench Excavation and Backfill

The same techniques recommended for excavation in above section is also recommended for trench excavation.

Surface water runoff should be prevented from entering the excavation by berms, swales, or other methods. Water should not be allowed to accumulate and pond within trenches since it will accelerate softening of the subgrade and may result in unacceptable subgrade soils and need for subgrade stabilization such as over-excavation and backfilling or drying and compaction.

Excavation trenches should be sloped and or supported in accordance with all federal, state, and local ordinances protecting workers.

9.3.1 Pipe Zone Backfill

Backfill material for the pipe zone should be placed from 6 inches below the bottom of pipe to 1 foot above the top of pipe. This pipe zone material should consist of sand, gravel, or crushed rock, reasonably well graded from coarse to fine, and free from excessive clay, organic material, and other deleterious substances. The pipe zone backfill material should contain a maximum of 8 percent passing the No. 200 sieve, and a maximum particle size not exceeding 1 inch.

Pipe zone backfill should be placed and spread in layers simultaneously on both sides of the pipe, not to exceed 6 inches loose thickness and compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557. The contractor should select compaction equipment with weight and energy delivered to prevent pipe damage during backfill operations.

Areas where weak and soft soil are encountered during pipeline construction should be over-excavated a minimum depth of 1 foot below the proposed trench bottom and replaced with granular fill compacted to 90 percent of the maximum dry density, as determined by ASTM D1557. In lieu of over-excavation of weak or soft soils, the use of controlled, low-strength material is recommended with the approval of the Engineer Controlled, low-strength material is a fluid mixture of Portland cement, water, and fine aggregates or fly ash. The consistency of the material is similar to flowable grout, and the material is placed like concrete. The mixture should be designed for a 28-day compressive strength of 150 to 250 psi.

9.3.2 Trench Backfill

Excavated soils meeting the requirements for fill and backfill could be used as trench backfill materials.

In areas beneath paved roadways, backfilling trench above pipe zone with granular fill compacted to 97 percent of the maximum dry density as determined by ASTM D1557 with maximum 6-inch loose lifts is required. However, in other areas that are not sensitive to settlement, excavated soil and processed rock can be used as backfill material.

In addition to those previously mentioned, the following considerations must be taken into account:

- Backfill material shall be free of roots, debris, organic material, rock larger than 3 inches, or other deleterious objects to be unsuitable for use as trench backfill above the pipe zone.
- Trench backfill should be placed and spread in layers with 6-inch maximum loose lifts and should be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. Moisture conditions of the soil being compacted shall be within 2 percent of the optimum moisture content.

9.4 Micropile Testing

9.4.1 Pre-Production Load Tests

Due to the large quantity of micropiles, five test micropiles are recommended to be installed to cover the footprint of the residual processing facility and where boulders were encountered during soil boring, such as within 3 feet from borings BH-02, BH-03, BH-08, BH-13, and BH-15. The pre-production should avoid the production pile locations.

The five pre-production test micropiles should be load tested to minimum 2.5 times design load and then to failure if feasible. The test micropile can either be installed after the site is excavated to the proposed grade or installed from existing ground surface with the soil above the proposed micropile top being isolated from the micropiles such that no resistance from soils above the proposed micropile top will be developed during pre-production load tests. Production micropiles should not be installed until these five test micropiles are successfully installed and load tested to required capacity.

One of the five pre-production micropiles is recommended to be installed using a larger size of temporary casing outside of permanent casing so that there will be no bond between soils/boulders above bedrock and steel casing being developed during load test. This allows more accurate estimate the grout-rock bond strength. Additionally, for each pre-production micropile, two levels of strain gauges with two strain gauges at each level are recommended to be installed at the top and the bottom of the bond zone to differentiate resistance from grout-soil bond and grout-rock bond. The gauges and wires should be properly protected from damage during construction and placement of grout.

The final rock socket length of micropiles should be determined based on pre-production load test results.

9.4.2 Production Micropile Proof Testing

Five percent (5%) of production micropiles are recommended to be proof tested per FHWA-SA-97-070 (Armour, et al. 2000). The test load is recommended to be 1.67 times design load.

9.5 Drilled Shaft Construction Considerations

It is believed that conventional drilling equipped with rock augers and core barrel can be used to advance shafts through soil and partially weathered rock. Coring will be required to advance the shaft into bedrock. Drilled shaft installation must be carefully coordinated by the contractor with the normal operation of the reservoirs.

9.5.1 Drilled Shaft Inspection

A qualified geotechnical engineer familiar with the site conditions, design intent, and proposed construction methods, should provide a comprehensive inspection program for quality assurance. Inspection is critical to confirm the conditions upon which design recommendations are based, and to assure that drilled shafts are constructed in accordance with the design intent. In addition, unexpected or unusual conditions that may be encountered could be addressed and immediately resolved.

9.5.2 Non-Destructive Testing for Drilled Shafts

To verify drilled shaft integrity, all production shafts shall be tested using a pile integrity test (PIT) to confirm that shafts are constructed in accordance with specifications, and there are no voids within the drilled shaft concrete or soft zone that exists at the tip of the shaft.

The PIT is a low-strain integrity test, alternatively called sonic testing, pulse echo, or transient response. This equipment may be used to test concrete drilled shafts. The PIT can detect the presence and location of potentially dangerous defects such as cracks, necking, soil inclusions or voids, and can determine shaft length. The equipment and technique are well established, corresponding to ASTM D5882. The top of the shaft must be accessible to perform the PIT test.

9.6 Floor Slab and Pavement Construction

Subgrade should be thoroughly proof-rolled with approved construction equipment to detect any soft or unstable areas. Any wet and/or unstable soils present at the subgrade level during grading operations should be either scarified, aerated, and re-compacted or should be removed and replaced with suitable fill material. For pavement construction, it is very important that the final soil subgrade be properly sloped or crowned to help remove surface water that develops from precipitation. It is very important that the pavement should be constructed immediately after acceptable subgrade conditions have been achieved, because of potential subgrade softening from adverse weather conditions.

SECTION 10

Limitations

This geotechnical report has been prepared in accordance with generally acceptable engineering practice. It is intended for the exclusive use of the U. S. Army Corps of Engineers, Baltimore District, for the proposed Residuals Collection and Treatment Facilities at the Washington Aqueduct Dalecarlia Water Treatment in Washington, D. C.

Information contained in this report is limited, based on data obtained from limited boring logs that show subsurface conditions only at the specific locations and times indicated, and only to the depths penetrated. Subsurface conditions and water levels at other locations or depths may differ from conditions indicated at the boring locations. The passage of time may result in change in the conditions at the locations. If, during construction, subsurface conditions are found to vary from those described in this report, the geotechnical recommendations are not warranted to be valid.

This report includes both factual and interpretive information. Factual information is defined as objective data based on direct observations, such as boring logs and laboratory test results. Interpretive information or geotechnical engineering interpretation is based on engineering judgment or extrapolation from factual information. No warranties, explicit or implied are provided.

SECTION 11
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